1.0 INTRODUCTION

This report is submitted as an integral component of the application for a permit to construct and operate a Class I municipal solid waste landfill facility at the Jungo Disposal Site located in Humboldt County, Nevada. The Jungo Disposal Site is located approximately 30 miles west of Winnemucca, Nevada along Jungo Road as shown in ***Figure 1***. Jungo Land and Investments, Inc. (JLII) will be the landfill developer and operator.

This report (***Volume I***) includes the Report of Design, Monitoring Plan, Closure Plan, and Postclosure Plan and was prepared in accordance with Nevada Administrative Code (NAC), Sections 444.677 through 444.683. ***Volume II*** includes the site design and development drawings. The Plan of Operations is included in ***Volume III***.

The landfill is located on a 634-acre parcel that consists of Section 7 of Township 35N (T35N), Range 33E (R33E). The landfill disposal footprint encompasses 560-acres within Section 7, T35N, R33E. The site development will include a railyard for unloading waste containers transported by the rail, an office trailer, and a maintenance shop. Additional detail on the supporting facilities is provided in the Plan of Operations.

This Engineering Design Report is organized as follows:

- Section 1 – Introduction
- Section 2 – Report of Design
- Section 3 – Monitoring Plan
- Section 4 – Closure Plan
- Section 5 – Postclosure Plan

The Engineering Design Report is divided into the following three volumes:

- Volume I includes the text, tables, and figures for the various reports, and supporting appendices consisting of data, test results and engineering calculations;
- Volume II includes the site development drawings; and
- Volume III includes the Plan of Operations.

2.0 REPORT OF DESIGN

The Report of Design was prepared in compliance with NAC, Sections 444.680 and 444.681. The Report of Design provides a description of the physical setting, and therefore, Section 2.1 also discusses the site location with respect to the location restrictions specified in NAC, Section 444.678.

2.1 Physical Setting

2.1.1 Site Location, Zoning and NAC Location Restrictions

The Jungo Disposal Site is located at the southern end of Desert Valley approximately 30 miles west of Winnemucca along the south side of Jungo Road as shown in ***Figure 1***. Regionally, the site is located in an arid, relatively flat, desert basin bound by the Jackson Mountains to the west and northwest, the Antelope Range to the southwest, Alpha Mountain to the south, and the Eugene Mountains to the southeast. The site vicinity is sparsely vegetated with greasewood.

The site is located on a 634-acre parcel consisting of Section 7 of Township 35N, Range 33E of the Mount Diablo Baseline and Meridian. The Jungo Disposal Site is bounded by Union Pacific Railroad property to the northwest and elsewhere by publicly-owned land administered by the Bureau of Land Management (BLM). ***Figure 2*** shows the property parcel in relation to the railroad and surrounding property sections.

The Jungo Disposal Site and surrounding property are zoned M-3 – Open Land Use. An M-3 designation allows for conditional commercial uses, such as landfill disposal operations, provided such uses are approved by the Humboldt Planning Commission (HPC). The HPC issued JLII a Special Use Permit in April 2007 that allows the site to be developed as a municipal waste disposal site.

NAC 444.678 specifies location restrictions for Class I landfills. The Jungo Disposal Site satisfies these requirements as follows:

- **NAC 444.678 – General**:
  - NAC 444.678 (1) The site design includes all-weather access to the landfill including an access road surfaced with aggregate. The rail unloading area will include paved areas and areas surfaced with aggregate to provide dust control and all-weather access.
  - NAC 444.678 (2) and (3) The landfill design includes containment systems, controls, and monitoring systems that will prevent uncontrolled migration of landfill gas, control leachate, and prevent degradation of groundwater.
  - NAC 444.678 (4) The site has soil available that is workable and compactable for use in covering the refuse.
  - NAC 444.678 (5) The disposal site also conforms with Humboldt County's land use planning.
  - NAC 444.678 (6) The nearest public highway (Interstate 80) is more than 30 miles from the site.
  - NAC 444.678 (9) The nearest surface water body is more than 14 miles from the site. The landfill is located within 100-feet of the uppermost groundwater aquifer. However, to prevent degradation of the groundwater aquifer, the landfill design incorporates extensive protective measures consisting of low-permeability

containment systems, conservatively designed leachate control system, and landfill gas control systems. These protective measures are described in Section 2.3.

- **NAC 444.6783 – Airport Safety**: There are no airports within 10,000 feet of the site.
- **NAC 444.6785 – Floodplain**: The site is not located within a floodplain. The site is located within a desert basin where precipitation temporarily collects in shallow depressions until it evaporates or infiltrates into the underlying soils.
- **NAC 444.679 – Wetlands**: The site is not located within a wetland area. The nearest wetlands in Desert Valley are more than 25 miles to the north along Bottle Creek Slough.
- **NAC 444.6791 – Fault Areas**: The site is located in a region that is underlain by a thick sequence of sediments that are at least 6,000 feet thick and do not show any surficial evidence of faulting (i.e. scarps). There are no mapped faults within 200 feet of the landfill. The nearest quaternary fault to the site is located in the Eugene Mountains at a distance of approximately 3 miles from the site.
- **NAC 444.6793 – Seismic Impact Zones:** The Jungo Disposal Site is located within a seismic impact zone, which is defined as a location that has a 10 percent probability of exceedance in a 250-year period of experiencing a seismically induced peak ground acceleration of 0.1g or greater. As required by NAC 444.6793, the Jungo Disposal Site is designed to withstand the peak ground acceleration without damaging environmental containment systems and controls, including the liner and cover systems. Seismic impacts are evaluated in Section 2.3.
- **NAC 444.6795 – Unstable Areas:** The Jungo Disposal Site is not located in an area that is considered geologically unstable, such a landslide prone areas, karst terrane, or excessively soft soils that could result in foundation failure. The site soils are expected to experience consolidation under loading by refuse. However, the landfill is designed to accommodate the settlement without adversely affecting the liner system as discussed in Section 2.3.

Additional detail on location restrictions is provided in the Plan of Operations.

2.1.2 Climate and Hydrology

The site is located in an arid region, where precipitation is controlled primarily by the rain-shadow effects imposed by the Sierra Nevada range located 150 miles to the west. The Jackson Mountains located on the west side of Desert Valley, cause a similar orographic effect, but of a lesser magnitude (Berger, 1995).

Precipitation results primarily from thunderstorms in the summer, and snow and rain in the winter. The mean annual precipitation is estimated to be approximately 8 inches. The mean annual precipitation in Winnemucca located 30 miles to the east (1897-2006) is 8.3 inches. Three different sources (Western Regional Climate Center; World Climate.com; and Berger, 1995) provide mean annual precipitation values ranging between 7.97 inches to 9.1 inches for a precipitation gauge at the Jungo-Meyers Ranch, located approximately 4 miles west of the Jungo Disposal Site. This precipitation gauge was measured from 1968 to 1986.

Temperatures in the summer months occasionally exceed 100° F. Winters are cool with temperatures often below 0°F. Based on data from Rye Patch Reservoir located 14 miles to the south, evaporation from free water sources is approximately 48-inches per year (Cohen, 1965). The prevailing wind direction in Desert Valley is toward the west-southwest.

The 25-year, 24-hour storm event is estimated to be 1.62 inches (NOAA, 2006).

2.1.3 Topography and Drainage

The Antelope Range, Alpha Mountain, and Eugene Mountains form the topographic divides at the southern end of the Desert Valley near the general location of the Jungo Disposal Site. The low-points of these topographic divides ranges from approximately 4,250 feet msl to 4,400 msl.

The valley floor is relatively flat. At the southern end of the valley, the elevations generally range from 4,180 feet msl to 4,155 feet msl and slope from the southeast to the northwest. At the Jungo Disposal Site, the elevations range from a high of 4,177 feet msl at the southeast corner of the property to a low of 4,172 feet msl at the southwest corner of the property.

Desert Valley is a 1,032-acre hydrographic sub-area within the Black Rock Desert Region hydrographic basin. Streams from the surrounding mountains are ephemeral and rarely discharge to the valley floor and instead infiltrate through the upper coarse alluvial fans or evaporate (Berger, 1995). Precipitation or snow melt on the valley floor accumulates in localized depressions until it infiltrates or evaporates.

At the Jungo Disposal Site, these shallow depressions are on the order of several inches deep. In the event of intense storms, it is possible that localized depressions may fill and then sheet flow to the next depressions located to the north or west. This is consistent with the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) (2007), which estimates that ponding may occur locally to depths of 6 to 12 inches.

Large alkali flats are located in larger depressions that are approximately 2 miles west and north of the Jungo Disposal Site with surface elevations ranging from 4,162 to 4,164 feet msl. These alkali flats are located in an area identified by Berger (1995) as containing hard-pan, or low-permeability clays and silts, which impede infiltration.

2.1.4 Geology

The following sections describe the regional geologic conditions and the site-specific geologic conditions based on subsurface explorations completed as part of the initial site characterization program.

2.1.4.1 Regional Geology

Desert Valley lies within the Basin and Range Geomorphic Province, which is characterized by north-south trending uplifted mountain ranges adjacent to down-dropped valleys or basins. Desert Valley is a north-south trending structural basin with a relatively flat valley floor that is approximately 55-miles long and 12-miles wide. Mountain ranges provide topographic boundaries at the edges of the valley floor. The Jungo Disposal Site is located in the southernmost portion of Desert Valley. This end of Desert Valley is bound to the west by the Jackson Mountains, to the southwest by the Antelope Range, and to the east by the Eugene Mountains. ***Figure 3*** shows the regional geologic map.

The lithology of the area is comprised of two major types - consolidated rock and basin fill. The rock is found in the surrounding mountains and underlying the valley basin fill sediments. Predominant rock types include Tertiary-age volcanic flows, clastic sedimentary rocks from the Jurassic or Triassic ages, and Permian-age or older volcanic rocks. These formations are generally considered to have low permeability.

At the base of the mountains, alluvial fans consisting of eroded sediments from the mountains occur. These cone-shaped deposits contain coarse sediments typically deposited by stream and debris flows.

The alluvial deposits range from partly consolidated to unconsolidated fill material. The Older Alluvium unit, consisting of poorly sorted, subangular to subrounded sand to cobbly gravel, has been identified to occur along several range fronts, high on the alluvial fans. Toward the valley floor, this unit underlies the Younger alluvium of the valley floor. The Older Alluvium grades finer toward the valley center and becomes partially consolidated.

In addition to the coarser-grained alluvial fan sediments, the basin contains aeolian, lacustrine, and volcanic deposits. During late Pleistocene time, Desert Valley was inundated by ancient Lake Lahontan, which reached depths of nearly 200 feet in Desert Valley. The contact between Older Alluvium and Younger Alluvium is typically drawn at the elevation of the highest Lake Lahontan terrace. The Younger Alluvium, located on the valley floor and beneath stream channels, includes Pleistocene and Recent lake sediments, shoreline deposits, stream deposits, and aeolian deposits. The total depth of the sediments in the basin is estimated to be 6,000 to 7,000 feet in the vicinity of the Jungo Disposal Site (Berger, 1995).

Older Alluvium, consisting of poorly-sorted, subangular to subrounded sand to cobbly gravel, generally occurs along the range fronts and grades finer toward the toes of the alluvial fans. Older Alluvium is primarily dissected alluvial fan deposits that are coarser than the Younger Alluvium found on the valley floors and in stream channels.

2.1.4.2 Site Geology

An initial site characterization program was completed to evaluate the site-specific geologic conditions, hydrogeologic conditions, and engineering properties of the underlying soils. This initial characterization program consisted of the examination and logging of the surficial soils, the completion of five borings to depths of 100 to 145 feet below ground surface (bgs), and the completion of a geotechnical laboratory testing program.

The site soils are mapped as Younger Alluvium and classified as Boton Playas described as unconsolidated alluvial sediments (USDA, NRCS, 2006). Boton soils consist of volcanic ash and loess over lacustrine deposits. The surficial soils are relatively uniform and consist of silty fine sands, classified as a silty sand (denoted as "SM") in accordance with the Unified Soils Classification System (USCS).

Five borings were completed using hollow-stem auger drilling methods under the observation of a Golder engineer who logged the soil samples and recorded groundwater conditions. The locations of the borings are shown in ***Figure 4***. Soil samples were collected at 5-foot intervals using a combination of standard split spoon samplers, modified California samplers, and Shelby (thin-walled) tubes.

The standard split spoon samples and modified California samples were obtained in accordance with the Standard Method for Penetration Test and Split Barrel Sampling of Soils as described in ASTM D1586. This sampling method consists of driving the split spoon sampler a distance of 18 to 24 inches into undisturbed soil. The number of blows required to drive a standard split spoon sampler the final 12-inches is known as the Standard Penetration Resistance "N", which provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Because drilling mud was not used during the drilling program, caution should be used in interpreting N values in cohesionless soils below the groundwater surface. Soil samples obtained for consolidation characteristic testing were obtained by pushing a three-inch outer diameter thin-walled tube into the undisturbed soil in accordance with the Standard Practice for Thin Walled Tube Sampling of Soils as described in ASTM D 1587.

All samples were placed in air-tight sample bags or sealed directly in the thin-walled tube to minimize moisture loss during transport to the laboratory. Soil samples were classified in accordance with Golder technical procedures and the Unified Soil Classification System.

The four borings located near the corners of the property were converted into groundwater monitoring wells (MW-1 through MW-4). The boring completed in the middle of the site was abandoned by backfilling the boring annulus with a cement-bentonite grout.

Summary boring logs and monitoring well completion logs are included in ***Appendix A***. Section 2.1.4.2.1 summarizes the subsurface geologic conditions observed. Section 2.1.4.2.2 summarizes the geotechnical engineering properties measured from the laboratory testing program.

2.1.4.2.1 Site Soil Conditions

The subsurface soils consist of interbedded sands, silts, and clays. ***Figures 5 and 6*** illustrate the subsurface lithology to a depth of 100 to 145 feet bgs, which is summarized below:

- **Upper Silty Sands**. The uppermost soils are predominately silty fine sands with occasional thin lenses of silt. These soils occur at the ground surface and extend to depths of approximately 35 to 40 feet bgs.

- **Upper Silty Clays and Clayey Silts**. A 10- to 18-foot thick layer of primarily silty clay and clayey silt underlies the uppermost silty sands.

- **Middle Sands**. At a depth of 55 to 60 feet bgs, the borings encountered predominately sands that are interbedded with silty sands and thin lenses of silts and silty clays. This soil zone was observed to be 18 to 30 feet thick.

- **Lower Clay and Clayey Silt**. A 12- to 20-foot thick clay layer was first encountered at a depth of 70 to 80 feet bgs. The upper portion of this layer is generally comprised of highly plastic and compressible clay, while the lower portion consists of low to moderately plastic clay.

- **Lower Sand and Silty Sand**. The deepest boring penetrated the lower clay and clayey silt zone at a depth of 115 feet and encountered interbedded sands, silty sands, and thin lenses of silt to the full depth of the boring at 145 feet.

As an approximate percentage of the lithologic section, clean sands comprise 10 percent, silty sands comprise 40 percent, silts comprise 10 percent, silty and sandy clays comprise 30 percent, and highly plastic clays comprise 10 percent.

2.1.4.2.2 Geotechnical Properties

Representative soil samples, collected from the borings during the investigation, were submitted to the laboratory for the following analyses:

- Moisture-density;

- Atterberg Limits;

- Grain-size distribution;

- Consolidation; and
- Consolidated-Undrained (CU) triaxial shear strength (with pore pressure measurements).

The key geotechnical properties are summarized below.

- SPT blow counts (N-Values, uncorrected) in the sands above the water table range from approximately 16 to 20 per foot in the upper 5 to 10 feet and generally increase to 40 at a depth of 45 to 55 feet. Based on the blow-counts, the sands are considered to be dense.
- The moisture content of the upper silty sand layer ranged from 12 to 18 percent. The dry density of the upper silty sand ranged from 92 to 106 pounds per cubic foot (pcf).
- The dry density of the silty clays ranged from 86 to 89 pcf. The dry density of the highly plastic clay was measured to be 59 pcf in two samples.
- The Plasticity Index of the silty clay (CL) layers ranged from 20 to 30. The Plasticity Index for the highly plastic clay (CH) was measured between 56 and 72.
- Consolidation tests indicated that the soils are generally normally consolidated. In some cases, the soils might be slightly over-consolidated, which may be related to the recent regional decline in groundwater levels and the older groundwater declines that occurred after ancient Lake Lahontan dried up. The primary compression index (C_c) for the silty clay and silt (CL-ML) was measured to be 0.16 to 0.26 with an initial void ratio of 0.9 to 1.0. The primary compression index (C_c) for the highly plastic clay (CH) was measured to be 0.66 to 0.71 with an initial void ratio of approximately 1.87. The lowest coefficient of consolidation (C_v) values were generally measured at between 0.01 to 0.08 ft/day, which corresponded to the high plasticity clays and some of the low plasticity clays.
- CU-triaxial shear strength tests measured effective stress strength parameters for the silty clays that can be defined by a friction angle of 26 to 27 degrees and cohesion of 1,500 to 2,100 psf. The measured effective stress strength parameters for the highly plastic clays can be defined by a friction angle of 19 to 21 degrees and cohesion of 800 to 975 psf.

Appendix B includes a summary of laboratory tests that were completed as part of the initial site characterization.

2.1.4.2.3 Summary

The initial site characterization indicates that the site is underlain by interbedded sands, silts, and clays. Four soil sequences were identified throughout the site in all five borings and included an upper silty sand, an upper silty clay and clayey silt, a middle sand, and a lower clay and clayey silt. One boring extended to a depth of approximately 145 feet encountered a fifth soil sequence consisting of a lower sand and silty sand. The base of the landfill, as described in Section 2.3 will be founded in the upper silty sand. Groundwater was first encountered in the middle sand layer at a depth of 58 feet. The upper silty clay layer occurs between the base of the landfill and first occurrence of groundwater.

The initial site characterization also indicated that the soils are normally consolidated, the silty clays and silts are moderately compressible, and that the highly plastic clay at a depth of approximately 80 to 90 feet is highly compressible.

As described in Section 2.3, the compressive characteristics of the underlying soils pose a significant constraint to the height and weight of refuse that can be placed on the liner. Excessive settlement of the foundation could result in adverse drainage grades on the landfill. Due to the critical aspect of these geotechnical properties, additional geotechnical borings will be completed prior to the construction of the base containment system as follows:

- A minimum of six borings will be installed for each module (each module is approximately 55-acres in area). After the explorations are completed for the first one or two modules, the number of borings may be increased or decreased based on the variability of the observed subsurface conditions. For the five borings that were completed for the initial characterization, the soil conditions were relatively consistent.
- The borings will be installed to depths of at least 200 to 300 feet.
- The underlying silts and clays will be sampled for further consolidation testing.

The above geotechnical investigations and testing will be used to confirm the current lithologic and geotechnical model. If these investigations indicate differing conditions, the lithologic and geotechnical model will be updated and the landfill design modified if necessary. Potential landfill design modifications may include changes to the drainage grades on top of the base liner or potential changes in the landfill heights. All design changes will be submitted the Nevada Department of Environmental Protection for review and approval prior to implementation.

2.1.5 Hydrogeology

2.1.5.1 Regional Hydrogeology

The partially-consolidated to unconsolidated basin-fill deposits in Desert Valley comprise the primary water-bearing unit. The deposits generally function as a single, heterogeneous aquifer rather than one with defined, contiguous fine-grained aquitards layered between coarser-grained water-bearing units. Most shallow groundwater occurs in unconfined conditions, while groundwater found at depth below finer-grained deposits occurs in semi-confined conditions (Berger, 1995).

Most groundwater recharge in the Desert Valley basin occurs as precipitation that falls on the mountains surrounding the basin. The primary mechanisms for recharge are for the rainfall or snowmelt to infiltrate exposed weathered and/or fractured bedrock or for the runoff to percolate through the coarser-grained alluvial fan deposits. With most recharge occurring at the higher elevations, groundwater at the eastern and western valley margins primarily flows from the higher elevations downgradient toward the center of the basin. Additional groundwater recharge does occur in the subsurface from the Quinn and Kings River Valleys located in the northern portion of the basin (Berger, 1995).

A groundwater divide bisects the Desert Valley basin from east to west. The location of the divide has been shown to migrate over time in response to changes in groundwater elevations, but has generally remained in the central to southern-central area of the basin located north of the Jungo Disposal Site. Groundwater on the northern side of the divide flows to the north toward the Quinn River and discharges out the northwestern side of the basin at Pine Valley. Groundwater on the southern side of the divide flows to the southwest and likely exits the basin near the Antelope Range (Berger, 1995). The Jungo Disposal Site is located on the southern side of the groundwater divide.

Ranges of horizontal groundwater hydraulic conductivities have been estimated by calculating average values for the different lithologic units encountered in the upper 180 feet of saturated basin-fill deposits in Desert Valley. Using the lithologic data, areas of the basin were roughly categorized as having horizontal conductivities either greater than 50 feet/day (ft/day) or less than 50 ft/day (Berger, 1995). Most of the basin was estimated to have a horizontal conductivity less than 50 ft/day (<1.7×10^{-2} cm/s), while some smaller areas of the basin were estimated to have conductivities greater than 50 ft/day. The Jungo Disposal Site is located in an area identified by Berger (1995) to have horizontal hydraulic conductivities less than 50 ft/day.

Prior to 1985, groundwater withdrawal in the Desert Valley basin occurred primarily for agriculture and irrigation, with lesser amounts for domestic and livestock use. In 1985, significant mining and associated dewatering operations began in the northern portion of the valley on the northern side of the groundwater divide at the Sleeper Mine. Basin-wide groundwater elevations measured in 1991 demonstrated that groundwater elevations in the basin had been affected by the mine dewatering. In general, elevations measured in 1991 (while mine dewatering was occurring) were lower than those from "pre-development" (late 1950's to early 60's) conditions. ***Figure 7*** compares groundwater flow directions in 1991 with estimated pre-development conditions in 1975. These 1991 measurements also showed that the groundwater divide had migrated toward the south. The Desert Valley basin groundwater elevations range from approximately 4100 to 4130 feet mean sea level (msl) (Berger, 1995).

Appendix C includes an evaluation of historical groundwater levels for the Desert Valley Basin. The data review determined that, in the vicinity of the Jungo Road site, groundwater levels have declined approximately 10 feet over the past 30 years. The decline is attributed to past and current groundwater withdrawal for agricultural use and mine dewatering. As such, with continued agricultural groundwater use and other development-related uses, groundwater elevations in the vicinity of the site will continue to decline, and therefore, they are not likely to exceed pre-development levels (i.e., approximately 1975 levels) and indeed may not rise beyond present day levels.

The data review presented in ***Appendix C*** also indicates that groundwater levels in the basin are not impacted by seasonal changes including periods with above average precipitation. ***Figure 8*** shows the historical trends in groundwater elevations for wells within the southern portion of the basin in comparison to annual precipitation values. ***Figure 9*** shows the locations of these reference wells. As indicated in ***Figure 8***, groundwater levels did not respond to several above-average rainfall years occurring in the late 1990's (1996, 1997, 1999, and 2001). To the contrary, groundwater levels continued to decline an additional two to three feet. Basin development and long-term groundwater use patterns (e.g., groundwater extraction for irrigation) appear to be a more significant factor in groundwater elevation change than annual precipitation.

2.1.5.2 Site Hydrogeology

In soil borings completed during the site investigation, first groundwater was encountered at approximately 58 to 60 feet bgs in the middle sand/silty sand layer, above a layer of low to high plasticity clay. Four of the five borings were converted to groundwater monitoring wells. The well locations are shown on ***Figure 4*** and the well construction details are provided in ***Appendix A***. Quarterly depth-to-water measurements have been taken in the wells since their installation in January 2007. These measurements indicate that groundwater occurs at elevations similar to those recorded in the initial soil borings. Therefore, first-encountered groundwater occurs under unconfined, water-table conditions, consistent with the regional hydrogeologic model.

As described above, water-levels in the basin have decreased, and in the area of the site have declined approximately 10 feet over the past 30 years. Current depth to groundwater at the site is approximately

58 to 60 feet bgs. Therefore, assuming a return to 1975 groundwater levels, the highest anticipated groundwater levels at the site are estimated to be approximately 50 feet bgs.

A groundwater contour map of the site has been prepared using groundwater elevations measured in the site wells (***Figure 10***). Based on these measurements, groundwater flows toward the southwest, consistent with the basin flow net prepared by Berger (1995). The gradient is estimated to be 0.0003.

Water-level measurements collected in between January 2007 and November 2007 have exhibited no seasonal variation. The maximum change in elevation during 2007 is less than one foot. In April 2007, a pressure transducer and datalogger were installed in well MW-1 to allow for continuous recording of groundwater levels to further evaluate any potential seasonal or other short-term variation.

Rising head slug tests were conducted in each well on February 2, 2007 to determine the hydraulic conductivity of the middle sand and silty sand. With these data, hydraulic conductivities were calculated for each well. To determine a hydraulic conductivity for the site, the geometric mean of the four individual well conductivities was calculated. As such, the hydraulic conductivity at the site is estimated to be 1.2×10^{-4} cm/s. The slug test data is presented in ***Appendix D***.

2.1.6 Seismicity

As defined by NAC 444.6793, the site is located within a seismic impact zone, which means the site is located in an area with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material will exceed 0.10g in a 250-year period.

Quaternary faults within a 10-mile radius of the site tend to be limited in length (10 km or less), and therefore, these faults have a limited capacity to generate large earthquakes. The nearest Quaternary fault is the Eugene Mountains Fault located approximately 3 miles southeast of the site. The nearest significant fault is the Western Humboldt Range fault zone located more than 20 miles southeast of the site.

The USGS database (2002) indicates that for the site (Latitude $40^0 55'$, Longitude $118^0 20'$) a peak ground acceleration (PGA) of 0.28g (in bedrock) is estimated for an earthquake event with a 2 percent probability of exceedance in a 50-year period. This is equivalent to an event with a 10 percent probability of exceedance in a 250-year period. Therefore, the landfill containment systems and environmental controls are designed to withstand an earthquake event resulting in a PGA of 0.28g without compromising the integrity of the containment systems and environmental controls. Section 2.3 describes these seismic impact evaluations.

2.2 Landfill Capacity and Site Development

2.2.1 Refuse Quantities and Landfill Capacity

The Jungo Disposal Site will serve as a regional disposal site for portions of Northern California and possibly Humboldt County. Although waste from Humboldt County is currently disposed of in the Humboldt County Regional Landfill, should the Humboldt County Commission so desire, the Jungo Disposal Site will accept local waste. Any refuse from Humboldt County will be delivered to the site in refuse hauling trucks. Refuse from Northern California will be delivered to the site by rail. Refuse from Northern California will comprise more than 95% of the waste stream, which is estimated to be up to an average of 4,000 tons/day.

The site will accept only municipal solid waste (MSW). Typically, MSW from Northern California is processed to remove recyclable or compostable materials including selected metals, plastics, and greenwaste. In addition, a screening program exists to remove hazardous waste before it is loaded into waste containers. The screening program is described in the Operating Plan.

Figures 11 and 12 illustrate the landfill base grading system and the final refuse fill geometry, respectively. The maximum refuse thickness is 200 feet at the center of the landfill. The maximum refuse height extends approximately 200 feet above the surrounding grades at the center of the landfill.

The disposal volume is approximately 106 million cubic yards. Based on an estimated in place effective density of 1,100 pounds/cubic yard (pcy), the landfill has a refuse capacity of approximately 58.5 million tons. Effective density is defined as the weight of disposed refuse divided by the total volume occupied by refuse and soil cover. For initial planning, it assumed that approximately 600,000 tons of refuse will be disposed annually. Accordingly, this disposal rate would result in a projected life of 97 years. The projected life will decrease as the disposal tonnages increase.

The base grades have been designed to maximize the separation between the bottom of the liner system and groundwater. The minimum separation distance is approximately 25 feet at the sumps. The average separation distance will be approximately 37 to 38 feet following base settlement induced by refuse loading (Section 2.3.4.1). Section 2.3 describes the containment systems and controls used to protect the underlying groundwater from potential impacts of leachate and landfill gas.

The excavation will generate a total excavation volume of approximately 12 million cubic yards of soil, of which approximately 6 million cubic yards will be used to construct the liner system and the final cover system. Approximately 6 million cubic yards of soil are available for daily and intermediate soil cover use, which requires a refuse to soil cover ratio of approximately 16:1. As described in the Plan of Operations, measures will be incorporated to limit soil usage for refuse disposal through the extensive use of Alternative Daily Cover (ADC) materials and a reduced soil cover thickness. For example, to conserve landfill airspace, alternative daily cover materials such as tarps or post-processing residue from recycling operations may be used as daily cover.

2.2.2 Site Development

The site development is illustrated in the landfill design drawings provided in ***Volume II***. The landfill disposal boundary is located 100 feet from the west, south, and east property boundaries. The disposal boundary is located 200 to 300 feet from the north property boundary to allow the development of a railyard for unloading waste containers.

As shown in ***Drawing 3 (Volume II)***, the landfill will be developed with 10 modules measuring approximately 56 acres each in area, where each module will contain a sump to collect and remove leachate. The site will be developed in phases with phases consisting of the construction of approximately 20 to 30-acres of the base liner. The landfill will also be closed in phases as the site is developed. Drawings 10 through 16 illustrate the landfill development, including base liner, refuse fill, and final cover construction at 2, 10, 25, 50 and 75 years following initial site development. During site development, areas that reach final grade will be allowed to consolidate and settle for a minimum 5-year period prior to final cover construction to reduce post-closure settlement impacts on the final cover. The final closure phase is an exception where the final cover will be constructed within 6-months of the placement of the last refuse shipment.

2.3 Containment System and Environmental Controls

The Jungo Disposal Site is designed with an extensive system of low-permeability containment systems, high-permeability leachate controls to minimize leachate head on the liner, and a landfill gas collection and disposal system to control landfill gas. The containment system design and environmental controls include the following enhanced landfill gas and leachate control features to provide additional groundwater protection:

- A high capacity leachate collection and removal system LCRS on top of a composite liner system. The high capacity will limit maximum leachate build-up to a fraction of an inch and thereby reduce the leakage potential of leachate.
- Additional pipes will be incorporated in the LCRS system that can be tied to a gas control system. This allows the potential to develop a vacuum on top of the liner to minimize the potential for the migration of landfill gas through the liner.

The following sections describe details of the containment systems and controls, and the engineering analyses used to support the landfill design.

2.3.1 Liner Design and Base Grading

The base liner system consists of a prescriptive composite liner system, in accordance with NAC 444.681, comprised of the following components from top to bottom on the floor of the landfill (see ***Drawing 4, Volume II***):

- 1-foot-thick operations soil layer;
- 1-foot thick gravel blanket for the LCRS with a permeability of 1 cm/s or greater;
- central leachate collection piping within each module to provide redundant leachate capacity;
- 16-oz geotextile cushion;
- 60-mil high-density polyethylene (HDPE) geomembrane; and
- 2-foot thick compacted low-permeability soil liner with a permeability (k) less than or equal to $1x10^{-7}$ cm/s.

On the side-slopes, the base liner system is comprised of the following components from top to bottom:

- 2-foot-thick operations soil layer;
- Geocomposite drainage layer (geonet with geotextile heat-bonded to both sides) for the LCRS;
- 60-mil HDPE geomembrane; and
- 2-foot thick compacted low-permeability soil liner ($k \leq 1x10^{-7}$ cm/s).

The base grading plan is shown in ***Figure 11***. The landfill will be divided into 10 modules with each module measuring approximately 56 acres each in area. The modules will be oriented in a north-south direction and the base grades are designed to minimize excavation depths and thereby maximize separation between the top of the composite liner system and groundwater. The floor of the landfill is graded at two percent toward the center of each module. The flowlines within each module extend from a center ridgeline oriented in an east-west direction and slope at a 1 percent grade to the north or south perimeter of the landfill. At the center ridge-line, the high end of the LCRS flow-line is near the existing ground surface. The maximum excavation depth is approximately 32 feet at each sump. Accordingly, the excavation plan results in the following groundwater separation distances:

- A minimum of 26 feet at the sump locations;
- A maximum of 58 feet at the center of the landfill; and

- An average separation distance of approximately 42 feet.

Based on the subsurface explorations described in Section 2.1.4, a generalized lithologic sequence indicates that the site is underlain by interbedded sands silts and clays. Two relatively low-permeability silt and clay layers were observed in each of the five borings, which appear to be relatively consistent across the site. The upper most silt and clay layer occurs at a depth of approximately 35 feet and 50 feet bgs, and therefore, provides another low-permeability barrier layer between the landfill and groundwater, which occurs at a depth of 58 feet bgs.

The existing site soils will not meet the permeability requirements for the low-permeability soil liner. Therefore, either suitable clay soils will be imported, or the on-site soils will be admixed with bentonite to produce a soil liner material with a permeability of 1×10^{-7} cm/s or less. Typically, an admixture of 5 to 9 percent of sodium bentonite by dry weight is required to produce a soil material that will meet this permeability requirement. The admixture percentage depends on the initial composition of the soils and the source of the bentonite. For the admix alternative, a bentonite admix design will be completed as part of the final drawings and technical specifications that will be prepared prior to the construction of each liner phase. In addition, construction quality assurance testing requirements will be established to verify that permeability requirements are achieved.

Construction Quality Assurance (CQA) will be completed during the liner construction activities to ensure that the construction complies with the liner design plans and specifications. Following each liner construction project, a certification report will be prepared and submitted to provide documentation that the construction activities were completed in accordance with the design plans and applicable federal and state regulations. A Nevada registered civil engineer will supervise CQA activities and certify the report.

Typical CQA activities will include, but are not limited to the following:

- Verification of the low-permeability soil materials including material quality, thickness, and compaction;
- Verification of the LCRS gravel including material quality and thickness;
- Observation and inspection of the geosynthetic materials for conformance with the engineering plans and specifications;
- Conformance testing of soil and geosynthetic materials;
- Documentation of construction procedures, and identification and resolution of construction problems; and
- Preparation of a CQA report providing documentation that the closure activities and construction complied with the project plans and specifications.

Once site development commences, the Jungo Disposal Site will also pursue, in consultation with NDEP, an alternative liner design that substitutes an encapsulated geosynthetic clay liner (GCL) in place of the 2-foot thick, low-permeability soil liner. An encapsulated GCL consists of GCL layer bounded by two geomembranes. The alternative liner design will include a leakage equivalency evaluation, slope stability evaluation, and a pilot field trial to verify performance. A work plan detailing the evaluations and field trial will be prepared and submitted to NDEP for review and approval. It is anticipated that the field trial will be completed within the first 5 years of the commencement of waste disposal.

2.3.2 Leachate Collection and Removal System (LCRS)

The landfill liner system design includes a blanket LCRS (***Drawing 4, Volume II***) that has a high hydraulic capacity that is designed to collect leachate while minimizing leachate head build-up on the liner. The maximum leachate head on the liner is estimated to be only a fraction of one-inch, which is considerably less than the 12-inch (30 centimeter) maximum depth allowed by NAC 444.681. The leakage potential of a liner system is reduced by decreasing the potential head build-up on the liner system.

On the floor of the landfill, the LCRS will consist of a one-foot thick gravel blanket with a 6-inch diameter HDPE drainage pipe located on the center of the flow-line of each module (***Drawing 4***). Leachate collected within each module will be conveyed to a 2-foot deep, gravel filled sump measuring approximately 40 feet by 40 feet in plan area. Liquids will be extracted from an HDPE riser pipe using either submersible pumps or a pneumatic pump system.

Based on very conservative estimates of leachate generation, the LCRS on the floor has a factor of safety of more than 200 for hydraulic capacity. Similarly, the side-slope drainage geocomposite drainage layer has a minimum factor of safety of approximately 50 for hydraulic capacity. Section 2.4.3 further describes the design hydraulic capacity of the LCRS. The LCRS design is very conservative in that the design leachate generation rates are expected to significantly exceed actual leachate generation rates. Golder's experience with leachate generation in landfills located in arid regions indicates that very little leachate will be generated at the Jungo Disposal Site during operation.

Extracted leachate will be used for dust control over constructed, lined modules. In the event that the collected leachate exceeds the dust control needs, the excess leachate will be re-circulated within the landfill. However, such recirculation volumes are expected to be very small with a negligent impact on the moisture content of the waste or depth of leachate head on the liner.

Leachate pipes will be designed to withstand the weight of the refuse without crushing or buckling. HDPE pipes with a size-dimension ratio (SDR) of 11 or less can readily withstand the loads imposed by 200 feet of refuse.

2.3.3 Landfill Gas Control

A gas control system will be used to collect and dispose of landfill gas. At a minimum, the gas control will comply with Federal New Source Performance Standards (NSPS) and Emission Guidelines and require a Title V Permit (40 CFR Part 70 and NAC 445B) prior to operating the gas controls. Conceptually, the landfill gas system will consist of a system of horizontal and vertical gas wells, HDPE collection and header pipes, and condensate sumps. Initially, landfill gas will be disposed of using flares. A Waste-To-Energy (WTE) system may be used to dispose of gas and generate electricity if such a system is determined to be feasible for the Jungo Disposal Site.

Refuse gas wells will be installed as the refuse is placed, or alternatively drilled into the refuse after refuse placement. Operation of the gas control system will not occur until there is sufficient amount of methane to operate a flare disposal system. For landfills that receive 12 to 20 inches of annual precipitation in the western U.S., this typically requires 1 to 2 million tons of refuse in place and a minimum of 2 to 4 years of decomposition. Due to the arid climate of the Jungo Disposal Site, a longer time period may be required before sufficient gas is generated for flare operations.

As a further groundwater protective measure, perforated gas extraction pipes will be incorporated in the LCRS layer to allow gas withdrawal from above the liner system. This will further reduce the potential of gas migration from the liner system.

2.3.4 Liner Engineering Evaluations

To ensure the liner performs as intended, engineering evaluations described in the following sections were completed to evaluate foundation settlement, slope stability, leachate drainage capacity, drainage, and closure design.

2.3.4.1 Base Settlement

The placement of refuse changes the stresses acting on the foundation soils, which will result in settlement of the soils supporting the liner system for portions of the landfill. This settlement will tend to result in flatter drainage grades along the liner system in the future. The analyses presented in this section evaluate the magnitude of the calculated settlements and the resulting impact on the future drainage capacity of the LCRS.

A preliminary settlement model was developed for the initial landfill design. The model used the five general lithologic layers described in Section 2.1.4 and interpolated similar conditions to depths of 250 to 275 feet beneath the landfill. The model considered one-dimensional vertical stresses, which likely exaggerates the estimated vertical stress differences (and calculated settlement) between the sump and the top of the side-slopes where the differential loading is the greatest.

Preliminary foundation settlement calculations included in ***Appendix E*** indicate a maximum settlement of between 9 and 10 feet in the center of the landfill and 4.5 feet at the sumps. The resulting post-settlement drainage grades are estimated to average 0.8 percent along the flowlines. The critical post-settlement grades occur between the sump and the top of the side-slope crest where the differential loading on the base is the greatest. In this location (area within approximately 300 feet of the sump), the preliminary estimated post-settlement grades are approximately 0.2 percent. As indicated in Section 2.3.4.3, this still provides relatively large factors of safety for hydraulic capacity.

Due to the need to maintain positive drainage on top of the liner for leachate control, base settlement is a critical design feature for this landfill. Therefore, additional borings and consolidation testing will be completed for each module as described in 2.1.4.2.3. In addition, the settlement model will be refined as part of the final design of each module and will consider two-dimensional vertical stress distribution. Depending on the findings of the additional geotechnical borings and further settlement modeling, it may be prudent to modify the landfill design to either locally steepen the base grades or flatten the side-slopes of the refuse fill (or both). In addition, depending on the revised settlement estimates, it may be prudent to implement settlement monitoring along the portion of the base with the greatest differential loading to verify the settlement model (along approximately the outer 300 feet of the floor). Substantive design changes will be submitted the Nevada Department of Environmental Protection for review and approval prior to implementation.

2.3.4.2 Slope Stability

Slope stability evaluations were completed to verify adequate stability under static and design seismic conditions. The primary failure mode of concern is the potential failure along the liner system, which generally has relatively low interface shear strengths. Potential failure of the underlying foundation soils was also considered.

Slope stability was evaluated using the computer program SLIDE (V. 3.047), which uses a two-dimensional method of slices and limit equilibrium methods to calculate factors of safety. The program was used to search for the failure plane with the lowest factor of safety.

Key assumptions common to the foundation and refuse slope stability analyses are summarized below.

- The shear strength of the refuse was modeled by a linear failure envelope represented by an internal angle of friction of 30 degrees and a cohesion of 200 pounds per square foot (psf), which is within the range of refuse strength parameters reported by Singh and Murphy (1990). These parameters are close to the values recommended by Kavazanjian (2001), which presents a refuse shear strength model with an internal friction angle of 33 degrees with a minimum shear strength of 500 psf.

- The unit weight of the total waste fill mass was assumed to be 70 pcf, which is a typical value for the unit weight of the refuse. Total weight is higher than effective density because it includes the weight of cover materials. This unit weight is conservative for a site that will extensively use ADC to minimize soil cover use.

- The critical liner interface is expected to occur between the compacted clay and textured HDPE geomembrane. The design interface shear strength was assumed to be defined by an effective friction angle of 12 degrees with no cohesion. Based on Golder's experience in performing interface shear strength tests on liner materials, this design interface shear strength is expected to be conservative. Interface direct shear strength testing will be completed once the low-permeability soils are identified as part of the final liner design plans. Interface direct shear testing will be completed as part of the Construction Quality Assurance Program to ensure that the minimum design liner interface shear strength is achieved.

- Seismic stability was evaluated using the simplified seismic design procedure developed by Bray et. al. (1998). The design PGA is 0.28g for bedrock. Attenuation of the ground motions in the overlying soils was conservatively ignored. It is common to apply a 10% reduction of the PGA to reflect attenuation in soils overlying bedrock. Based on computed yield accelerations, permanent displacements were estimated for the design seismic event.

- The underlying sands were assumed to have a shear strength corresponding to a friction angle of 30 degrees. The low plasticity clay was assumed to have a friction angle of 27 degrees based on the result of laboratory testing. The highly plastic clay was assumed to have a friction angle of 20 degrees based on the results of laboratory testing.

- The underlying clays were assumed to be drained during loading. Preliminary analyses indicate that the clays will reach 80 percent consolidation within 4 years of loading. Therefore, the development of excess pore pressures should be negligible.

Appendix F includes the results of the slope stability analyses. For potential failure along the liner, a static factor of safety of 1.9 was computed, and permanent seismically induced displacements were calculated to be less than 1 inch. Displacements of up to 6 to 12-inches along the liner system are generally accepted as being within the tolerance limits of liner systems without resulting in adverse damage.

Potential failure of the foundation soils is not a critical failure mode since the shear strengths of the native soils are considerably higher than the assumed liner interface shear strength.

Liquefaction of the underlying soils during an earthquake is considered highly unlikely at the Jungo Disposal Site due to the depth of groundwater at 58 feet bgs. Liquefaction due to seismically–induced strong ground motions are generally observed in saturated sands and silty sands loc`ated within 40 feet of the ground surface. Confining stresses at greater depths significantly reduce the liquefaction potential. The additional weight of refuse on the underlying soils will further increase the confining stresses, and thereby further reduce the liquefaction potential.

2.3.4.3 Leachate Generation and LCRS Capacity

A very conservative leachate generation model was developed to conservatively size the hydraulic capacity of the LCRS. A conservative approach was used to provide an additional level of environmental protection relative to leachate management.

The model was developed using the computer program Hydrologic Evaluation of Landfill Performance (HELP). ***Appendix G*** includes details on the HELP modeling for the Jungo Disposal Site. The conservatively developed HELP model estimates a peak leachate generation rate of 75 gallons/acre/day (gpad) for the Jungo Disposal Site. This estimated leachate generation rate is very high for an arid site with only 8-inches of average annual precipitation. This level of leachate generation is comparable to modern, composite-lined landfills in Northern California with an average annual average precipitation of 25 to 30 inches. Golder's experience with landfills in arid regions is that they produce very limited to no leachate. The proposed Rawhide Landfill in Nevada, which is located in an area with approximately 6-inches of annual precipitation, estimated no leachate production.

The design of the LCRS consists of a high permeability gravel blanket draining a 2 percent grades toward perforated HDPE collection pipes. The HDPE pipes drain at a one percent grade toward the perimeter of the landfill.

The impact of base settlement is most severe in a direction perpendicular to the refuse slopes, which is in a direction parallel to the LCRS collection pipes. The settlement calculations indicate a post-settlement grade of between approximately 0.2 to 0.8 percent along these pipes. Settlement along the floor grades toward the LCRS pipes will be considerably less since the differential stresses and resulting differential settlements are less in the flow direction along the floor toward the pipes.

The maximum area draining to a single sump will be approximately 56 acres. Using the design peak leachate generation rate of 75 gpad, the maximum leachate flow drained by a single collection line is approximately 3 gpm.

The capacity of a 6-inch diameter HDPE pipe at a 0.2 percent grade servicing that flow is 135 gpm. Given potential base settlement and the maximum leachate generation rates, the resulting factor of safety for pipe capacity is more than 30 during operations. In addition, the peak leachate head depth on the liner is estimated to be well less than 0.25-inches. As indicated in ***Appendix G***, the hydraulic factors of safety for the blanket LCRS layer on the base and side-slopes are more than 200 and approximately 50, respectively.

2.3.4.4 Drainage Controls During Operations

Drainage controls will be implemented during site development to control surface water run-on and run-off. Surface water run-on will be prevented by the following measures:

- A 4-foot high perimeter berm will be constructed to prevent run-on from shallow (6-inch to 12-inch) ponding that may occur locally following intense thunderstorms.

- Temporary retention basins will be located adjacent to module excavations to collect precipitation that occurs within the landfill excavation footprint. Water will be pumped from the temporary basin to the perimeter of the landfill. Drawing 10 illustrates an example of such a basin.

Surface water run-off will be controlled by ditches and down-drains that will be sized to accommodate a 25-year, 24-hour storm event in accordance with NAC 444.6885. A shallow ditch will be included around the perimeter of the site. The perimeter ditch will promote the accumulation of water until it exceeds the ditch depth and sheetflows westward to the surrounding grades where it will accumulate in shallow depressions until it evaporates or infiltrates into the underlying soil.

2.3.5 Closure Design

A final cover system will be constructed over the waste at the Jungo Disposal Site as part of the closure activities. The final cover system is a prescriptive cover, in accordance with NAC 444.6891) consisting of the following components (***Drawing 8, Volume II***):

- A minimum 2-foot thick vegetative soil layer (infiltration and erosion layers);
- A geocomposite drainage layer (to collect and drain water infiltration below the vegetative layer);
- A 60-mil HDPE geomembrane layer (textured on both sides);
- A one-foot thick low-permeability soil layer ($k \leq 1x10\text{-}7$ cm/s); and
- A two-foot thick foundation layer.

The above cover system provides a low-permeability barrier that has permeability less than or equal to the base liner system. HELP modeling of the cover system indicates that a negligible amount of water will infiltrate through the cover. HELP analyses for the closed conditions are summarized in ***Appendix G***.

The Jungo Disposal Site will pursue an alternative Evapotranspirative (ET) final cover design once the landfill is in operation. An ET cover typically consists of 3 to 5 feet of soil that stores infiltration and then releases it through evapotranspiration. Based on Golder's experience with ET covers, the site climate and soil types appear suitable for an effective ET cover system. The alternative ET cover design will include supporting soil laboratory testing and unsaturated flow modeling. If the modeling results indicate that ET cover is equivalent or superior to the prescriptive cover system, then a field trial will be constructed on portions of the landfill that have achieved final grades. A work plan detailing the laboratory testing, modeling, and field trial program will be prepared and submitted to NDEP for review and approval.

2.3.5.1 Final Cover Grading

Figure 12 shows the final cover grades for Jungo Disposal Site landfill. The final cover grades reach a maximum elevation of 4,172 feet mean sea level (msl) and maintain a maximum side-slope inclination of 4H:1V (horizontal to vertical). To facilitate drainage and minimize erosion, 25-foot wide benches are incorporated into the side-slopes a maximum of every 50 feet vertically. The top surface will be graded at 5 percent to accommodate postclosure refuse settlements and maintain positive drainage.

2.3.5.2 Erosion

Final landfill slopes will be inclined no steeper than 4H:1V. Minimum final surface slopes will be 5 percent. To mitigate potential wind and water erosion, the vegetative layer thickness was increased from one foot to two feet.

As part of the closure activities, the integrity of the final site face will be maintained by the placement of a vegetative layer to provide erosion control. The slopes will be revegetated with desert grasses that are suitable for the non-irrigated areas of the Desert Valley basin.

An erosion analysis was completed for the slopes using the Revised Universal Soil Loss Equation program, RUSLE Version 1.06 (United States Office of Surface Mining and Reclamation, 1998). The analysis results indicate an estimated maximum soil loss for the proposed final grades of 0.03 inches/year which is less than an average of approximately 1–inch over a 30 year postclosure period. The erosion soil loss analysis is presented in ***Appendix H***.

2.3.5.3 Postclosure Cover Settlement

Settlement analyses were performed to evaluate the impact of postclosure settlement on the final cover grades. Refuse settlement typically exhibits a large, rapid, initial settlement rate referred to as primary settlement, which is followed by a long-term, progressively decreasing, settlement rate that is referred to as secondary settlement. Primary settlement generally occurs within weeks to months of the initial refuse placement. However, secondary settlement occurs for many years as waste materials decompose and compress.

The calculated postclosure settlements assume that primary settlements are complete prior to closure, but secondary settlements will continue throughout the entire 30-year postclosure monitoring period. As indicated in ***Appendix I***, the postclosure grades following settlement will be greater than three percent, which is sufficient to promote positive drainage from the cover.

2.3.5.4 Cover Veneer Slope Stability

The stability of the cover system considers the potential occurrence of a failure within the final cover components. This failure mode is primarily a function of the interface strengths of the cover materials and the maximum final slope inclinations. Static stability analyses were completed using an infinite slope analysis and verified by the computer program XSTABL (v. 5.2.02). Yield accelerations were determined using the computer program XSTABL (v. 5.202). XSTABL uses two-dimensional, limit-equilibrium methods to evaluate stability.

Evaluation of the stability of the cover components was based on the following assumptions:

- The maximum cover grade was assumed to be 4H:1V (maximum slope in between benches). Following closure, settlement will reduce the slope height and inclination, which will tend to increase slope stability with time;
- The critical interface occurs between either the vegetative soil layer/geocomposite drainage layer or the geocomposite/textured geomembrane layer. Based on a shear strength data base prepared by Golder's Geosynthetics Laboratory (***Appendix F***), the critical shear strength parameters were assumed to be represented by an internal friction angle of 23 with no adhesion. At low normal loads, the interface shear strength between the textured geomembrane and

underlying low-permeability soil layer is expected to be greater than 23 degrees based on Golder's data base.

- The simplified seismic design procedure by Bray et. al. (1998) was used to estimate seismic displacements for the cover system using the design PGA of 0.28g.

The factor of safety for static conditions is calculated to be 1.7 (***Appendix F***). The results for the design MCE seismic loading conditions indicate seismically induced permanent displacements of less than 4-inches. Based on current engineering practice, a maximum allowable seismically induced permanent displacement of 6 to 12 inches is acceptable for modern landfills located in a seismic impact zone.

2.3.5.5 Surface Water Controls

Surface water controls will be installed on the final cover system to control surface water run-off and minimize erosion of the cover system. Drawing 7 illustrates a conceptual surface water drainage plan for the Jungo Disposal Site at closure. Surface water will be controlled by ditches on the slope benches, berms on the top-deck of the landfill, and down-drains along the side-slopes. All surface water controls are sized to accommodate the 25-year, 24-hour storm event (NAC 444.6885).

A shallow ditch will be included around the perimeter of the site. The perimeter ditch will promote the accumulation of water until it exceeds the ditch depth and diffuses laterally to the surrounding grades where it will accumulate in shallow depressions until it evaporates or infiltrates into the underlying soil.

3.0 . MONITORING PLAN

Environmental monitoring will be completed during landfill development and following closure and will include groundwater monitoring, leachate monitoring, and landfill gas monitoring. Surface water monitoring will not be completed because there is no nearby surface water body. However, storm water monitoring will be completed in accordance with NPDES requirements.

The monitoring plan focuses on detecting potential releases from the landfill. However, there are no nearby off-site groundwater wells that would be impacted by a release from the site. There are no municipal water wells within 10 miles of the site. The nearest groundwater well is used for agricultural purposes and is located more than one mile northeast of and upgradient from the landfill site.

3.1 Groundwater Monitoring

This groundwater monitoring plan includes a description of the existing groundwater monitoring network (MW-1 to MW-4) as well as the conceptual expansion of the groundwater monitoring network as the area of waste placement extends laterally from the northwestern corner of the Facility. The sampling and laboratory procedures proposed for groundwater monitoring at the Facility to ensure monitoring results representative of background and downgradient water quality are detailed in this plan. Included with these procedures are the required monitoring parameters, frequency of monitoring, and QC specifications for both field and laboratory activities. The proposed methods for data verification and statistical evaluation also are described. This monitoring plan complies with the requirements of the Nevada Administrative Code (NAC) Chapter 444 Section 683 and the Code of Federal Regulations (40 CFR), Parts 258.51 and 258.53.

The protocols outlined in this monitoring plan will serve as the basis for implementing the Facility's groundwater detection monitoring program, and for any subsequent assessment or corrective action monitoring, should it be required. This plan is designed to be protective of human health and the environment. As additional data is obtained through future site investigations and routine monitoring, or if changes in regional groundwater conditions are identified, it may be appropriate to revise and/or update this plan to ensure that it provides an effective and efficient means of monitoring groundwater quality in the vicinity of the disposal Facility.

A description of the current groundwater monitoring network and its appropriateness is provided in this section. Establishment of this initial monitoring network has been based on research of the Desert Valley basin hydrogeology and field investigations conducted at the site. The strategy for augmenting the network to maintain its effectiveness as the Facility expands over time is discussed.

The proposed Facility is located in the southern portion of Desert Valley, south of the groundwater divide that bisects the valley. The regional groundwater flow direction in the vicinity of the site has been documented to be toward the southwest (Berger, 1995) which is consistent with recent data collected from the site. In the five exploratory borings drilled at the site in January 2007, groundwater was encountered at a depth of approximately 60 feet below ground surface (4,105 to 4,115 ft MSL). No perched saturated zones were encountered above this depth. The thickness of the first-encountered water-bearing zone ranged from approximately 10 to 30 feet. Groundwater was found to occur most frequently in sand and silty sand/sandy silt units.

Four of the five exploratory borings completed at the site were converted to groundwater monitoring wells which comprise the current monitoring network (MW-1 through MW-4). The wells are located at the four corners of the site boundaries. Based on hydrogeologic conditions observed at the site, the wells were constructed to monitor appropriate locations and depths and to yield representative groundwater samples from the uppermost aquifer. A summary of well construction details is provided in ***Table 2***. The well network and direction of groundwater flow are shown in ***Figure 10***.

Samples collected from these wells, prior to construction of the Facility, are providing background groundwater quality data, both upgradient of the proposed Facility site and at the boundary of the waste unit. Once waste placement commences, well MW-2 will be designated as a hydraulically-upgradient background well based on the groundwater flow direction determined from the field investigation (Section 2.1.5.2 Report of Design). Wells MW-1, MW-3, and MW-4 will be designated as detection monitoring wells, as they are located adjacent to the downgradient boundaries of the waste management unit.

The interim groundwater monitoring system is established in accordance with NAC 444.7483. As required, the system at the proposed Facility currently consists of a sufficient number of wells installed at appropriate locations and depths to yield samples of groundwater from the uppermost aquifer. Details of the site hydrogeological setting, including lithology and stratigraphy of the basin deposits, estimates of hydraulic conductivity and effective porosities, and the direction / velocity of groundwater have been described previously in Section 2.1.

3.1.1 Monitoring Network Expansion

As the waste modules at the Facility are filled over time, the monitoring network will be expanded to maintain the effectiveness of the monitoring program. The proposed waste fill sequence plan specifies initial waste placement in the northwestern corner, with subsequent cell construction and filling occurring from the northwestern corner toward the southwestern corner. Cell construction also will occur toward the east to allow for stable and efficient waste filling, though the primary direction of waste placement will proceed toward the south. As cells are constructed on the western side of the site, groundwater monitoring wells will be installed along the western boundary, directly adjacent to and downgradient of the waste modules. A well spacing of approximately 950 feet is proposed as shown on ***Figure 13***.

Once the westernmost modules have been completed, cell construction has been proposed to continue from north to south, adjacent to and east of the existing cells. By completing phases from the west toward the east, future cells are continually constructed directly upgradient of existing cells and the existing detection monitoring well network located along the western site boundary. This strategy maximizes the use of the initial western-boundary well network.

As waste cell construction progresses from the west to east, detection monitoring wells will be installed along the southern, downgradient boundary of the Facility. Specifically, monitoring wells will be installed directly downgradient of the leachate sumps, which will be located along the southern edge of the Facility. A leachate sump is the most likely location for landfill leakage, due to the flow of leachate toward the sump and the accumulation of leachate in the sump. As shown on the conceptual final monitoring network in ***Figure 13***, this strategy will result in a well spacing interval of approximately 950 feet along the southern and western (downgradient) boundaries of the Facility.

3.1.2 Groundwater Sampling procedures

An accurate representation of background and downgradient water quality will be obtained from the samples from the monitoring wells which were installed in accordance with NAC 444.7483. The methods and procedures for groundwater sampling are described below. These procedures for groundwater sampling are designed to provide consistent and reproducible results and ensure that the overall objectives of the monitoring program are achieved. As required by NAC 444.7484 (1), documentation for the sampling and analytical program is hereby placed in the operating record, and includes procedures and techniques for: 1) sample collection, preservation, and shipment; b) analytical procedures; c) chain-of-custody control; and d) quality assurance (QA) and quality control (QC). The following documents have been used as guidelines for the development of these procedures:

- *Procedures Manual for Groundwater Monitoring at Solid Waste Disposal Facilities* (EPA-530/SW-611, August 1977)
- *RCRA Groundwater Monitoring Technical Enforcement Guidance Document* (OSWER 9950.1, September 1986)
- *Standard Guide for Sampling Groundwater Monitoring Wells* (ASTM, D 4448-85a)
- *Standard Practice for Decontamination of Field Equipment Used at Non-radioactive Waste Sites* (ASTM, D 5088-90)
- *Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)* (ASTM, D 4750-87)
- *Test Methods for Evaluating Solid Waste: Physical/Chemical Methods* (EPA SW-846, Base Manual [3rd edition, November 1986], through Update III [June 1997]).

Pursuant to requirements of NAC 444.7484 (subsections 2 and 3) the procedures outlined below are appropriate for groundwater sampling and will accurately measure all required constituents. These procedures are considered protective of human health and the environment.

3.1.2.1 Sample Collection

Sample collection procedures include equipment cleaning, well purging, and sampling.

3.1.2.1.1 Equipment Cleaning

Before the sampling event, all equipment that is placed in the well or comes in contact with groundwater is disassembled and cleaned thoroughly with detergent water, and then steam cleaned or rinsed with de-ionized water. Any parts that may absorb contaminants, such as plastic pump valves, bladders, etc., are cleaned or replaced.

For electric submersible pumps used for purging wells, all external pump surfaces and the discharge tube are steam cleaned prior to lowering the pump into the well casing. An aqueous solution of Liquinox (phosphate-free detergent), followed by de-ionized water, is then run through the pump and discharge tubing to clean internal surfaces. Water is prevented from draining back though the pump by an in-line check valve located immediately above the pump.

3.1.2.1.2 Well Purging

Before sampling, standing water in the casing and sand pack is purged from the monitoring well using either a positive displacement polyvinyl chloride (PVC) hand pump, a portable or dedicated electric submersible pump, a PVC or polyethylene bailer, a centrifugal pump, a dedicated pneumatic bladder pump, or a peristaltic pump. Field measurements for pH, specific conductance, turbidity, and temperature are recorded at casing volume intervals during purging on water sample field data sheets. The field measurements are used as indicator parameters to determine when a representative sample can be taken. Purging is generally performed until stabilization (± 10 percent variation) of the indicator parameters takes place. If a well dries during purging, it will be allowed to recharge for up to 24 hours; samples will be collected as soon as sufficient volume is available. If a well does not recharge sufficiently within 24 hours, the well will be considered dry for that sampling event.

Once detection monitoring commences, all purge water will be containerized on site pending analytical results. Purge water will then be disposed of in accordance with applicable local, state, and federal regulatory requirements.

3.1.2.1.3 Well Sampling

Groundwater samples are collected using a Teflon bailer, an individually sealed disposable polyethylene bailer, a dedicated electric submersible or pneumatic bladder pump, or in-line through a peristaltic pump with clean tubing. Wells are sampled in progression from "clean wells" to wells yielding poorer-quality water. The purpose of this procedure is to reduce the potential for cross contamination of wells by purging or sampling equipment.

Clean glass bottles of at least 40 milliliters volume fitted with Teflon-lined septa are used to collect samples for volatile organic analyses. These bottles are completely filled to prevent air from remaining in the bottle. A positive meniscus forms when the bottle is completely full. A convex Teflon®-lined septum is placed over the positive meniscus to eliminate air. After capping, the bottles are inverted and tapped to verify that they do not contain air bubbles. The sample containers for other parameters are filled, filtered as required, and capped.

If dissolved concentrations of metals are required, appropriate field filtration techniques are used. When using a bailer for sampling, a transfer vessel is filled with sample and fitted with a disposable 0.45-micron acrylic copolymer filter. Air pressure is applied to the transfer vessel forcing the sample through the filter; the filtrate is then directed into the appropriate containers. If a pump is used for sampling, the filter is placed in-line at the end of the discharge tubing and the filtrate directed into the appropriate containers. Each filter is used once and discarded.

3.1.2.2 Sample Preservation and Shipment

Sample containers and preservatives vary with each type of analytical parameter. Container types and materials are selected to be non-reactive with the particular analytical parameter tested. Sample preservatives used are consistent with regulatory guidelines and specified analytical methods.

All sample containers are labeled immediately following collection. Samples are kept cool with blue ice until received by the laboratory. At the time of sampling, each sample is logged on a chain-of-custody record, which accompanies the samples to the laboratory. Water samples are transported from the site by the sampler to a state-certified laboratory facility or to a secure interim shipping location.

Upon receipt of the samples by laboratory personnel, the chain-of-custody record is signed and released, and a unique sample identification number is assigned to each sample container. This number is recorded on the chain-of-custody record and is used to identify the sample in all subsequent internal chain-of-custody and analytical records. The manager of the subcontracted laboratory ensures that the holding times for requested analyses are not exceeded.

3.1.2.3 Sample Documentation

The following procedures are used during sampling and analysis to provide chain-of-custody control during sample handling from collection through storage. Sample documentation includes the use of the following:

- Water sample field data sheets to document sampling activities in the field
- Labels to identify individual samples
- Chain-of-custody record sheets for documenting possession and transfer of samples

3.1.2.3.1 Water Sample Field Data Sheets

In the field, the sampler records the following information on a water sample field data sheet:

- Location
- Project number
- Client name
- Sample ID
- Name of sampler
- Regulatory agency
- Date and time
- Pertinent well data (e.g., casing diameter, depth to water, well depth)
- Calculated and actual purge volumes
- Purging equipment used
- Sampling equipment used
- Appearance of sample (e.g., color, turbidity, sediment)
- Results of field analyses (e.g., temperature, pH, specific conductance)
- Purge water containment
- General remarks, including well accessibility and integrity

The sampler signs the field data sheets.

3.1.2.3.2 Labels

Sample labels contain the following information:

- Project number
- Sample ID (e.g., well designation)
- Sampler's initials
- Date and time of collection
- Type of preservative used

3.1.2.3.3 Sampling and Analysis Chain-of-Custody Record

The sampling and analysis chain-of-custody record, initiated at the time of sampling, contains, but is not limited to, the well number, sample type, analytical request, date of sampling, and the name of the sampler. The record sheet is signed and dated by the sampler when transferring the samples. Custody transfers are recorded for each individual sample. The number of custodians in the chain of possession is kept to a minimum. A copy of the final sampling and analysis chain-of-custody record is returned to the sampling contractor with the laboratory analytical report.

3.1.2.4 Field Quality Assurance Procedures

The objectives of the field program are to generate monitoring data of the highest possible quality and to ensure that these data are defensible during review. In general, QA/QC protocols are based on published USEPA guidelines. Field QA/QC is further ensured by training requirements for all field technicians.

Field QA procedures are specified for each sampling event. Field QA typically includes documenting field instrument calibration, and collecting and analyzing trip blanks, field blanks, equipment blanks, and duplicate samples. The analysis of trip, field, and equipment blanks, prepared with organic-free water, are used to detect contamination introduced through sampling procedures, external field conditions, sample transportation, container preparation, sample storage, and the analytical process.

Trip blanks are prepared at the same time and location as the sample containers for a particular sampling event. Trip blanks accompany the containers to and from that event, but at no time are they opened or exposed to the atmosphere. Typically, one trip blank for volatile organic parameters will be included per sampling event.

Field blanks are prepared in the field so they are exposed to the ambient atmosphere at a specified monitoring point during sample collection to determine the influence of the external field conditions on sample integrity. Equipment blanks are prepared in the field to ensure that sampling equipment does not cross-contaminate water samples. Organic-free water is run through the properly cleaned or unused (if disposable) sampling equipment, collected and analyzed. One field blank or equipment blank for volatile organic parameters will typically be included per sampling event.

Duplicate samples are collected to assess sampling and analytical precision. For each sampling event including more than six wells, duplicate monitoring well samples will typically be collected at a frequency of 10 percent. Where possible, field duplicates are collected at sampling points known or suspected to contain chemical constituents of interest. Duplicates are packed and shipped blind to the laboratory for analysis with the samples from that particular event.

3.1.2.5 Monitoring Frequency

The monitoring program will include semi-annual monitoring of all wells, including both detection and background wells. Per NAC 444.7488, four independent samples are to be collected and analyzed for program constituents from each background and downgradient well during the first semi-annual sampling event. To evaluate potential seasonal changes in groundwater chemistry, this plan proposes collecting and analyzing four independent sample sets in 2007, prior to the placement of waste at the site. After these four quarterly samples have been collected and analyzed, the sampling and analysis program will resume a semi-annual schedule unless it is determined that additional background water quality data is required for the selected statistical method.

3.1.2.6 Groundwater Level and Total Depth Survey

Before each sampling event, the static water level will be measured in appropriate monitoring wells and piezometers. The water-level gauging will occur within a period of time short enough to avoid potential temporal variations in groundwater elevation. The monitoring wells are purged and sampled for chemical constituents after measuring water levels.

The water level in the wells and piezometers is measured with an electric sounder with cable markings stamped at 0.01-foot increments. The water level is measured by lowering the sensor into the monitoring well. A low current circuit is completed when the sensor contacts the water, which serves as an electrolyte. The current is amplified which activates an indicator light and audible buzzer, thus signaling when water has been contacted. A sensitivity control compensates for very saline or conductive water. The electric sounder is decontaminated by rinsing with a detergent solution then de-ionized water after each use. Depth to water is recorded to the nearest 0.01 foot on a water level data sheet. .The groundwater elevation at the monitoring well is calculated by subtracting the measured depth to water from the surveyed elevation of the top of the well casing.

Total well depth is measured in monitoring wells scheduled for sampling by lowering a probe to the bottom of the well and recording the depth. Total well depth, used to calculate purge volumes and to determine whether the well screen is partially obstructed by silt, is typically recorded to the nearest 0.1 foot on the water level data sheet.

3.1.3 Laboratory Analytical Procedures

The monitoring parameters and methods for analysis are detailed in this section.

3.1.3.1 Water Quality Parameters

Constituents to be monitored for, as listed in Appendix I of 40 CFR Part 258 and as required by NAC 444.7487, are presented in ***Table 3***. These constituents include VOCs and select metals. In addition to these required constituents, analysis for general water parameters also will be conducted (e.g., bicarbonate alkalinity, total dissolved solids, chloride). These additional parameters are useful for assessing the geochemistry of the groundwater and for identifying changes that may occur as a result of a release or regional conditions. The recommended analytical method for these constituents is included in the table.

As allowed by Section 7487, the list of routine constituents may be re-evaluated after a period of time to determine if any of these constituents should be removed from the list, should it become apparent that they are not reasonably expected to be in or derived from the waste units. Alternative inorganic parameters that provide a more reliable indication of a release also may be proposed.

Prior to waste placement, baseline water quality will be developed using the existing groundwater monitoring wells to characterize the initial water quality conditions. These wells will be monitored at least once for a wide range of organic analytes, including semi-volatile organic compounds (SVOCs), polychlorinated biphenyls (PCBs), organochlorine pesticides, organophosphorus pesticides, and chlorinated herbicides. This one-time sampling and analysis event will provide additional data regarding existing groundwater conditions in the vicinity of the site.

3.1.3.2 Methods

Water samples collected for compliance monitoring will be analyzed by a Nevada state-certified laboratory. Samples will be analyzed in accordance with accepted and approved analytical procedures. The analytical procedures shall have detection and/or reporting limits that are sufficiently protective and of human health and the environment. The following publications are the primary references for analytical procedures:

- *Methods for Chemical Analysis of Water and Wastes* (EPA 600/4-79-020, Revised March 1983)
- *Methods for Organic Chemical Analysis of Municipal and Industrial Wastewater* (EPA-600/4-82-057, July 1982)
- *Standard Methods for the Examination of Water and Wastewater*, APHA, AWWA, WPCF, 17th edition, 1989
- *Test Methods for Evaluating Solid Waste: Physical/Chemical Methods* (EPA SW-846, 3rd edition, November 1986)

3.1.3.3. Quality Assurance

Laboratory QA procedures are employed to ensure that results are accurate, precise, and complete so that the overall objectives of the monitoring program are achieved. Laboratory-specific procedures are included in the laboratory's QA manual, including the use of method blanks, surrogate spikes, laboratory control samples (and duplicates), and matrix spikes (and duplicates).

Method blanks are analyzed daily to assess the effect of the laboratory environment on the analytical results. Method blanks are performed for each parameter analyzed and are expected to be clean. The presence of the subject compound or analyte at a significant level indicates the potential for sample contamination.

Each sample analyzed for organic parameters contains surrogate spike compounds. The surrogate recovery is used to determine if the analytical instruments are operating within limits. Surrogate recoveries are compared to control limits established and updated by the laboratory based on its historical operation.

Laboratory control samples (LCS) and LCS duplicates are prepared and analyzed for each batch of samples to evaluate the accuracy and precision of the methods. A known amount of the subject analyte is spiked into a clean water sample; analysis for the subject analyte subsequently is conducted to assess the method accuracy. The recovery of the subject analyte must be within QC limits. If the LCS recovery does not pass, re-analysis of all samples in the batch should occur. A duplicate LCS is prepared and analyzed to assess the method precision.

Matrix spikes are analyzed at a frequency of approximately 10 percent. Matrix spike results are evaluated to determine whether the sample matrix is interfering with the laboratory analysis and provide a measure of the accuracy of the analytical data. Matrix spike recoveries are compared to control limits established and updated by the laboratory based on its historical operation.

Matrix spike duplicates are analyzed at a frequency of approximately 10 percent. Spike duplicate results are evaluated to determine the reproducibility (precision) of the analytical method. Reproducibility values are compared to control limits established and updated by the laboratory based on its historical operation.

Laboratory QC data will be reported with the analytical results. The review of QC data is an integral step in the data verification process and may identify potential laboratory errors or biases affecting the data.

3.1.4 Data Evaluation

The following activities are required to evaluate groundwater data collected from the monitoring network.

3.1.4.1 Data Review and Validation

Prior to entering data into the facility database and prior to conducting statistical evaluations, all analytical reports will be reviewed to verify that the reports are complete and correct. The use of proper QC measures should be verified. Any QC issues that occur and have the potential to affect the analytical results for site samples should be further evaluated prior to data acceptance. Re-testing may be a necessary step in data validation should a result appear suspect based on accompanying laboratory QC results or other data comparison.

3.1.4.2. Statistical Analysis

As required by NAC 444.7485, a statistical method will be used in the evaluation of groundwater monitoring data for each hazardous constituent. The amended Federal and State regulations provide a variety of statistical methods that may be used to evaluate water quality data. Selection of the most appropriate comparative methodology and data analysis cannot be performed until adequate background and monitoring information has been obtained. Therefore, the actual method used will be based on a review of the data set prior to the time that the statistical analysis is to be performed. Performance standards of the selected procedure will be in accordance with NAC 444.7485.

The number of samples collected to establish background data concerning the quality of groundwater will be consistent with the requirements of the selected statistical procedure. To establish a data set that will adequately characterize the background range of concentrations of constituents, a minimum of four sample sets are recommended to be collected from each well prior to placement of waste at the facility; however, certain statistical procedures may require eight data sets for the calculations. Sampling of the current monitoring network has occurred three times and is anticipated to continue on a quarterly basis through 2007.

3.1.4.3 Detection Monitoring

Detection monitoring is required at all Class I waste management units. The detection monitoring program for the proposed Facility will follow the requirements of NAC 444.7488. Comparisons of detection monitoring results to established background values will occur with each semi-annual monitoring event in order to determine whether a statistically significant increase in constituent concentrations has occurred in accordance with NAC 444.7485.

In the event a constituent in the monitoring program demonstrates a statistically significant increase, the following actions shall occur per NAC 444.7489.

- Within 14 days of the finding, JLII will place a notice in the operating record. The notice will indicate which constituents have shown increases. NDEP also will be notified of this action.
- If the increase cannot be demonstrated to result from a source other than the waste units, an assessment monitoring program shall be established within 90 days. The assessment monitoring program will be established in accordance with NAC 444.749 and will include at a minimum, sampling for all Appendix II constituents.
- Should JLII determine that the increase is not a result of a release from the waste unit, but rather another source (e.g. natural variation, sampling or laboratory error) then a report documenting such must be placed in the operating record within 90 days.

Results of the semi-annual detection monitoring program will be submitted semi-annually to the required agencies in an acceptable format.

3.1.4.4 Assessment Monitoring and Corrective Measures

An assessment monitoring program is established to evaluate an indication of an increase of one or more monitored constituents. Should an assessment program be necessary, it will be initiated per NAC 444.749. If results of the assessment monitoring indicate one or more Appendix II constituents are present at a statistically significant level above the standard for the protection of groundwater, the actions required under subsection 3 of NAC 444.749 will be taken. As required, an assessment of corrective action measures will be initiated within 90 days.

3.2 Landfill Gas Monitoring

Landfill gas migration from the landfill is unlikely due to the presence of a low-permeability composite liner system and the use of an extraction system to collect and remove gas from the landfill. Perimeter subsurface landfill gas monitoring and indoor structure monitoring will be conducted to verify adequate control of landfill gas.

Perimeter landfill gas monitoring will consist of quarterly sampling and testing of gas probes located at the landfill property boundary. The probes will be spaced at 1,000-foot intervals resulting in a total of 21 perimeter probes as shown in ***Figure 14***. Each landfill gas probe will consist of two nested probes located in the upper silty sands at depths of approximately 30 feet and 15 feet with each probe containing a 5 to 10-foot long screen (***Figure 15***). The underlying silty clay layer, which is described in Section 2.1.4.2, should inhibit downward migration of landfill gas in the unlikely event that gas migrates from the landfill. Indoor air monitoring in the office and shop structures also will be conducted quarterly.

The concentrations of combustible gas (as methane), oxygen, carbon dioxide, and the barometric pressure will be measured using appropriate field instrumentation. Prior to use, all field instrumentation will be calibrated properly according to the manufacturer's recommendations. A minimum of one probe casing volume will be purged using the instrument's sample pump. Meter readings will be allowed to stabilize for 30 seconds before recording the gas concentrations. Landfill gas monitoring will be conducted to verify that explosive gas content does not exceed the lower explosive limit (LEL), equivalent to 5 percent methane by volume, at the perimeter boundary. Structure monitoring will be conducted to verify that concentrations remain below the allowable upper limit of 25 percent of the LEL, equivalent to 1.25 percent methane by volume.

In the event methane is detected at a concentration greater than 5 percent by volume in the perimeter probes, or greater than 1.25 percent by volume in a landfill structure, steps will be taken to protect human health and the source of the methane will be investigated. Corrective measures will be implemented to reduce methane concentrations to acceptable levels.

3.3 Leachate Monitoring

Leachate quantities will be recorded on a weekly basis for each leachate sump. On an annual basis, leachate samples will be collected and analytical testing will be completed. The leachate samples will be collected as follows:

- One leachate sample will be collected per sump.
- For a given leachate sump, if the constituent and constituent levels remain relatively consistent for 5 annual sampling events, then future sampling frequencies may be reduced and/or eliminated.

The testing protocols will follow that identified for groundwater in Section 3.1.

4.0 CLOSURE PLAN

At closure the Jungo Disposal Site Landfill will measure approximately 560-acres in area and contain approximately 58 million tons (106 mcy) of refuse. ***Figure 12*** shows the proposed final cover contours. The maximum side-slope inclination is 4H:1V (horizontal to vertical); the maximum elevation is 4,375 feet mean sea level (msl), or approximately 200 feet above the surrounding ground surface. The top deck of the landfill will be graded at 5 percent to accommodate postclosure settlements and maintain positive drainage.

In accordance with NAC 444.6892, JLII will provide notice to the solid waste authority of the intent to close the landfill within 15 days of initiating closure. Landfill closure activities will begin within 30 days of the receipt of the final refuse shipment. Landfill closure will be completed within 180 days of the date closure activities are initiated, unless the solid waste management authority grants a schedule extension (NAC 444.6892.3).

4.1 Final Cover System

A final cover system will be constructed over the waste at the Jungo Disposal Site as part of the closure activities. The primary functions of the final cover system are to:

- Isolate the waste from the environment;
- Control odors, vectors and litter;
- Control surface water infiltration into the landfill;
- Control erosion and run-on (if any), and convey run-off to the surface water management system; and
- Control landfill gas.

The final cover system for the Jungo Disposal Site is a prescriptive cover consisting of the following components:

- A minimum 2-foot thick vegetative soil layer;
- A geocomposite drainage layer;
- A 60-mil HDPE geomembrane layer (textured on both sides);
- A one-foot thick low-permeability soil layer ($k \leq 1x10^{-7}$ cm/s); and
- A two-foot thick foundation layer.

The site will have a landfill gas collection system fully installed prior to closure. Therefore, closure construction requirements for the landfill gas collection system are limited to activities integrating the landfill gas extraction wells and piping into the closure cover design. Integration of gas controls with a closure cover system is routinely completed and standard conceptual design details are included in ***Volume II***.

Drawing 7 shows a conceptual drainage plan for Jungo Disposal Site at Closure. Drainage will be conveyed along the top deck and intermediate slope benches to down-drains located along the sides of the landfill. The down-drain pipes will be fitted with diffuser tees at the discharge ends to dissipate high velocity hydraulic energy before discharging to the perimeter channels. Run-off will then be conveyed off-site where it will eventually collect in shallow depressions until it evaporates or infiltrates into the underlying soils similar to the existing surface water conditions in the site vicinity.

Appendix J presents conceptual drainage calculations to verify that the above conceptual drainage facilities are designed to accommodate a 25-year, 24-hour precipitation event as required by NAC 444.6885.

4.2 Postclosure Land Use

The postclosure end use of the site will be undeveloped open space, which is consistent with surrounding terrain and land uses. The site is planned to be maintained as secured non-irrigated open space and the closed landfill will be designed to reduce health and safety impacts with proper site security fencing and access control.

4.3 Environmental Monitoring and Controls

Environmental monitoring and controls will consist of leachate monitoring, groundwater monitoring and landfill gas monitoring. The leachate sumps, groundwater monitoring network, and landfill gas monitoring network will be installed as the site is developed, and therefore, they will be fully installed prior to the closure of the last landfill cell. No additional environmental monitoring and control systems will be installed at closure. However, these systems will be operated and maintained as discussed in Section 5.

4.4 Closure Activities

4.4.1 Phased Closure

Closure will be completed in phases as the site is developed. Generally, closure activities will occur in areas where the final grades have been achieved for a period of at least 5 years. This will allow the largest waste settlements to occur prior to closure, and therefore, reduce postclosure settlement impacts. Based on this phased closure approach, the maximum extent of closure at any point in time is estimated to be 205-acres or less.

4.4.2 Site Security, Dismantling and Structure Removal

JLII will provide site security upon closure. Site security will include:

- Proper signs posted at all points of access;
- Access will be controlled by locked gates at all access points around the perimeter; and
- Fencing will be maintained around the entire site.

The operating facilities (office and maintenance shop) and operating equipment will be removed from the site at the time the final phase of closure is completed.

4.4.3 Final Cover Construction

A final cover will be constructed as part of the closure activities. The final cover, as described in Section 2.3.5 and 4.1, is a prescriptive composite cover system. Prior to completing closure construction, construction plans, technical specifications, and a construction quality assurance plan will be prepared and submitted to the Nevada Department of Environmental Protection (NDEP) for review.

Construction Quality Assurance (CQA) will be completed during the closure activities to ensure that the construction complies with the closure design plans and specifications. Following closure construction, a closure certification report will be prepared and submitted to provide documentation that the closure activities were completed in accordance with the design plans and applicable federal and state regulations. A Nevada registered civil engineer will supervise CQA activities and certify the closure report.

Typical CQA activities will include, but are not limited to the following:

- Verifying the materials, thickness and compaction of the foundation layer;
- Observation and inspection of the geosynthetic materials for conformance with the engineering plans and specifications;
- Conformance testing of soil and geosynthetic materials;
- Documentation of construction procedures, and identification and resolution of construction problems; and
- Preparation of a CQA report providing documentation that the closure activities and construction complied with the project plans and specifications.

4.5 Closure Cost Estimate

Table 4 summarizes the closure cost estimate representing the maximum closure costs at point during site development. Key cost assumptions include:

- Environmental controls are installed prior to closure;
- The maximum extent of closure at any point in time is 205-acres or less, including the first phase of closure; and
- The low-permeability soil layer will consist of on-site soils admixed with approximately 6 to 8 percent bentonite.

Closure funding will be based on the proportion of the volume of waste disposed to the final refuse volume. In addition, the closure funding will be established prior to the construction of each phase of the base liner system.

5.0 POSTCLOSURE PLAN

JLII will implement postclosure monitoring and maintenance of Jungo Disposal Site, which will be performed for a period of 30 years following closure. Postclosure activities will consist of the following:

- Groundwater, leachate, surface water, and landfill gas monitoring;
- Operation of leachate collection and disposal controls and the landfill gas collection and disposal system; and
- Inspection and repair of the final cover system and other environmental controls.

These postclosure activities are described in further detail in Sections 5.1 through 5.3. Section 5.4 summarizes the postclosure monitoring and maintenance cost estimate.

Prior to implementing final closure, the Jungo Disposal Site emergency response plan will be updated and include JLII emergency contact information, local emergency contact information, and NDEP emergency contacts. Written notification of unusual incidents or occurrences observed during inspections will be provided to NDEP regarding such events as; vandalism, fires, explosions, earthquakes, surface drainage problems; and other incidents involving or potentially threatening waste releases.

5.1 Monitoring and Sampling Activities

Monitoring and sampling activities include leachate, groundwater, and landfill gas. Sampling and analysis of groundwater and gas will be performed on a semi-annual basis. Leachate monitoring at the sumps will be performed at least semi-annually. If necessary, more frequent monitoring of leachate will be completed to ensure leachate is managed to prevent accumulation of leachate to a depth of more than one foot on the liner system.

Based on the landfill design, the groundwater monitoring plan (Section 3), and landfill gas monitoring plan (Section 3), the monitoring system will include the following:

- 10 Leachate Sumps;
- 12 Groundwater Monitoring Wells; and
- 21 Landfill Gas Monitoring Probes.

Postclosure groundwater monitoring and landfill gas monitoring will follow the procedures described in Sections 3.1 and 3.2, respectively, except that the frequency will be reduced from a quarterly basis to a semi-annual basis.

5.2 Operating Activities

A landfill gas extraction and disposal system will be operated until landfill gas generation rates no longer support the gas extraction and disposal system. For cost estimating purposes, it is assumed that landfill gas will be collected and disposed of using a series of landfill gas flares. Landfills generate methane gas, which in many cases can be used to generate electricity. JLII will investigate the feasibility of such Waste-To-Energy (WTE) uses during operations. However, until WTE is determined to be feasible for the Jungo Disposal Site, a flare disposal system is conservatively assumed in the postclosure monitoring and maintenance cost estimate.

Leachate will be monitored at the sumps, and if liquids pond to depths of about 6-inches or more, they will be extracted. If necessary, an approximately 0.5-acre, double-lined (HDPE geomembrane) evaporation pond will be constructed on north side of the landfill to accommodate leachate. The pond also will be used to dispose and evaporate landfill gas condensate.

An evaporation pond may not be required. Due to the arid environment, a very limited amount of leachate (or no leachate) is expected to be generated during operations, when leachate generation rates are the greatest. A study by the U.S. Environmental Protection Agency (USEPA, 2002) indicates that leachate generation in landfills typically reduces to 10 percent of the leachate generation rate during operations after about 4 years following closure. Within 10 years following, leachate generation rates are near zero. Therefore, leachate generation should be negligible for the Jungo Disposal Site following closure.

The site will implement closure in phases and JLII will be able to monitor the impact of closure on leachate generation rates during the site development. This will allow JLII to refine the leachate disposal needs prior to the closure of the last cell.

5.3 Postclosure Inspection and Maintenance Activities

Postclosure inspection and maintenance activities will include the final cover, the site drainage system, environmental controls, and security system as described below.

The final cover will be inspected semi-annually to confirm that the final cover continues to function as an infiltration barrier. Visual inspections will be performed by qualified personnel to verify the integrity of the final cover. The cover will be inspected for signs of settlement and subsidence, erosion, cracking or other items that could adversely affect the integrity and effectiveness of the final cover. Items requiring corrective action will be repaired as soon as feasible.

Some minor differential settlement is expected at every landfill. Minor settlement can create relatively small depressions on a landfill surface where water will pond. At the Jungo Disposal Site, repair of such ponds will be completed in one of the following ways:

- Small depressions will be filled with soil to promote positive surface drainage.
- Larger depressions in which the underlying geocomposite drainage layer is not positively drained will be excavated to remove the cover system components above the foundation layer. Additional foundation soils will be added as necessary to establish suitable drainage grades. The overlying cover components will be replaced using the existing cover materials or new materials as may be necessary. The replaced materials will be constructed in compliance with the original closure engineering plans, specifications, and CQA plan.

Appendix I presents the results of settlement analyses that were completed to evaluate the effects of postclosure settlement on the final cover grades. The results of these analyses indicated that the proposed grades are sufficient to accommodate the anticipated post-closure settlement and still provide adequate drainage. In addition, because the phased closure approach will be generally implemented in areas of the landfill where final grades have been established for at least five (5) years, postclosure settlement impacts on the final cover should be limited.

Additional inspection activities include:

- The vegetative cover will be inspected for signs of erosion, degradation, and areas that lack vegetative growth. Items requiring corrective action will be repaired as soon as feasible. The postclosure maintenance costs provided in Section 3.3 assume that reseeding will be completed for an average of 25 acres per year.

- The surface drainage controls will be inspected annually for evidence of damage, excessive erosion, settlement, and obstruction by debris. The effectiveness of the surface water drainage ditches will be maintained by keeping the ditches, down-drains, and culverts clear of debris, excess soils and excess vegetation. Repairs to the structures will be made if the inspections reveal excessive damage to the ditches, down-drains and culverts. In addition, regrading will be performed as necessary to maintain positive drainage.

- As part of the periodic sampling program, the groundwater wells, leachate riser pipes, and landfill gas probes will be inspected for damage. Well heads, locks, caps, sampling ports and/or tubes that appear damaged or excessively worn will be identified and replaced.

- All locks, gates, signs, and fences will be inspected on an annual basis. Any damage to the security system due to vandalism, trespassing, or natural wear and tear will be immediately repaired and/or replaced. Signs will be repainted or replaced on an as-needed basis to maintain their visibility.

5.4 Cost Estimate

Table 5 presents a 30-year postclosure maintenance cost estimate for Jungo Disposal Site. JLII will establish a postclosure maintenance fund prior to developing landfill disposal modules. The postclosure maintenance costs and fund are reviewed and updated annually.

The current cost estimate for postclosure maintenance of the Jungo Disposal Site is based upon information presented in this report and includes an area of approximately 350-acres, which is the largest area of the landfill under the 30-year postclosure area at one time. The following key assumptions were made in compiling these estimates:

- Environmental monitoring costs are based on the projected number of sampling points and testing described in the monitoring plan (Section 3) with semi-annual monitoring by a third party.

- On average, about 20-acres of the cover will require reseeding each year;

- Inspections are completed annually;

- Landfill gas operation and maintenance will occur throughout the entire 30-year postclosure period.

As indicated in ***Table 5***, the projected annual postclosure maintenance cost is approximately $417,000/year.